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PRESTRESSED CONCRETE GIRDERS, MANHATTANVILLE COLLEGE, PURCHASE, N.Y.

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STRUCTURAL DIVISION

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PRESTRESSED CONCRETE GIRDERS
MANHATTANVILLE COLLEGE, PURCHASE, N. Y.

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The 65 ft. prestressed concrete girders at Manhattanville College do not represent any spectacular advance in the use of prestressed concrete. They do, however, constitute the first major application of prestressing to building construction in this country and demonstrate a new system of tensioning and anchoring parallel wire cables with greater control and accuracy than was possible heretofore.

In one of the new buildings constructed for the Manhattanville College at Purchase, New York, an 8500 sq. ft. assembly hall and gymnasium had to be carried over the dining hall as seen in Figure 1. The original intention was to carry this assembly hall on six concrete encased structural steel girders each 3 ft. 6 in. deep, spanning 64 ft. and 18 ft. 8 in. centers. Shortage of steel during the construction period made the obtaining of the steel sections extremely difficult and the architect called upon the engineers to redesign the girders in ordinary reinforced concrete. The results of this redesign showed that the depth of the girders would have to be increased from the original 3 ft. 6 in. to 6 ft. 0 in. resulting in a general increase in height of the entire building of 2 ft. 6 in. and a consequent increase in cost if the desired ceiling height was to be retained. It was then decided to examine the possibilities of using prestressed concrete. The Preload Company was consulted and a prestressed concrete girder section only 4 ft. in depth was designed. This depth was only 6 in. greater than the original encased steel girder and 2 ft. less than the reinforced concrete girder.

The superimposed permanent loads amounted to 122 lbs. per sq. ft. and the live load to 85 lbs. per sq. ft. giving a total load per girder of 130 tons.

The prestressed concrete design for the six girders required a total weight of steel of only 8 tons as against the 72 tons of this critical material which would have been required had the structural steel girders been used. This 89% saving in the weight of steel required was quite impressive.

In order to avoid the payment of large royalties for the use of European patented systems of prestressing a method of tensioning and anchoring the prestressing wires, used in this country for several years for the vertical stressing of circular tank walls, was modified to suit the requirements of linear prestressing.

The system used is illustrated in Fig. 2 from which drawing it is seen that the 12 wires of the circular prestressing cable are each anchored individually by conical wedges in a 4" x 4" square 1 1/4" thick anchor plate after being all prestressed together by the hydraulic jack indicated by the dotted line. To minimize the dimensions of end bearing plates, precast concrete cylinders, 6" diameter x 6" long, with heavy helical reinforcing to absorb concentrated bursting stresses, were placed in the girders under each bearing plate. These cylinders are required to support the bearing plates at a compression stress of 3000 p.s.i.

Fig. 3 shows schematically how the cast-in-place floor slab, the top of which is flush with the top of the girders, is carried on each side of the girder by a small 14" x 8" beam spanning 7 ft. 7 in. between brackets on the prestressed girders. As an additional security dowels extend into the floor slab on each side of the girder top flange. The bracket blocks were precast as were the end blocks to which were anchored most of the prestressing cables. The remainder of each girder was cast in place.

Each entire girder could have been precast but its 28 ton weight precluded its handling with such equipment as the contractor had available on the site and the number of girders involved was not considered sufficient to justify the obtaining of special lifting plant.

DESIGN

5000 p.s.i. concrete was specified and by the use of high early strength cement this strength was attained in seven days. The girders were so designed that under working load no tension can occur in the concrete and the maximum compressive stress is 2073 p.s.i.

Table I shows the calculated top and bottom fiber stresses at mid span at the different stages of construction and under live load. The part of the table marked "A" shows stresses based on initial values of prestress before losses due to relaxation of concrete and steel. Part "B" of the table shows stresses based on final values of prestress after all relaxation has taken place. The actual stresses at any particular stage depend on the age of the concrete and the time that has elapsed since the last application of prestress and lie therefore between these two sets of values. The stresses "A" are probably nearly true until after application of the first phase of prestressing at the age of 7 - 10 days. The actual stresses tend towards values "B" thereafter.

To follow through Table I in detail it can be seen that when the first ten cables constituting the first stage of prestressing are tensioned, the result is a tension at mid-span in the top fiber of 33 p.s.i. and a compression in the bottom fibers of 1614 p.s.i. These values will tend to change with time to a compression of 85 p.s.i. in the top fiber and a compression of 1144 p.s.i. in the bottom fiber. When this stress change has been only partially effected, however, after a delay of about three weeks, the load of the newly-poured slabs is transferred from its formwork to the girders resulting in top and bottom fiber stresses that lie somewhere between a compression of 907 p.s.i. at the top and a compression of 384 p.s.i. in the bottom, and a compression of 1025 in the top and a tension of 86 p.s.i. in the bottom. Just after the taking up of this live load true stresses are probably approximately 950 p.s.i. compression at the top and zero compression at the bottom. The remaining 12 cables constituting the second stage of prestressing are now tensioned resulting in a bottom fiber compression at mid-span somewhere between 2157 and 3190 p.s.i., the acutal value was probably in the region of 2800 p.s.i. and this critical compression stress experienced at this stage of the construction was considerably in excess of any stress to which the girder would likely be subjected to in its later life under anything but extreme overload. As the remainder of the permanent load is applied to the girder, it can be seen from the table that the stress conditions improve until the normal top and bottom fiber stresses at mid-span under all permanent loads once all relaxations have taken place are respectively 1050 p.s.i. compression and 1386 p.s.i. compression or an almost uniform compression over the whole girder section. Application of the 85 p.s.f. live

load does not induce any tension in the girders and the maximum compressive stress is only 2073 p.s.i.

An important fact that emerges from the above, and which is generally typical of prestressed concrete, is that the girders are subjected to much higher stresses during construction when the concrete is younger and weaker, than they are under full design loads later on. They can, therefore, be well described as "self-testing."

The 460 ton total final prestressing force necessary in each girder was obtained by the use of 22 cables each of twelve 0.196 in. diameter high tensile wire grouped in a steel sheath around a central helical wire spacer. Fifteen of these cables were anchored at each end of each girder and 7 were anchored at different points along the top surface of the top flange (see Fig. 4).

The guaranteed ultimate tensile strength of the high tensile steel wire was 220,000 p.s.i. It was stressed initially to 150,000 p.s.i. and this dropped after relaxation of the concrete and steel to the design stress of 120,000 p.s.i.

The sloping up of the cables near each end of the girder had the theoretical advantage of making it possible to eliminate practically all diagonal tension and the practical advantage of relieving the congestion of anchorages at each end which would otherwise be present in these exceptionally heavily prestressed girders.

The 22 cables were each numbered and their order of tensioning was carefully specified in order to ensure that the resultant prestress force during all stages of the prestressing operations was as close as possible to its final desired position.

The elongation to be obtained on the tensioning of each cable was specified and this varied from two inches to 4-11/16 in. according to the length of the cable.

On application of the first phase of prestressing it was calculated that the girders would rise up from their forms at mid-span a distance of 5/8 in. which proved in fact to be the average measured upward deflection (it varied according to the girder from 1/2" to 3/4"). This close agreement between calculated and measured deflections was confirmation that an approximate modulus of elasticity of 5,000,000 p.s.i. for suddenly applied loads had been attained in the concrete at an age of between 7 and 10 days. This was considered very satisfactory.

CONSTRUCTION

The first stage was the precasting of the bracket blocks and end blocks and Fig. 5 shows one of these precast blocks being lowered into position on the formwork for a girder. Two of the precast end blocks can be seen already in position.

Parallel with this operation the prestressing cables were being manufactured at first floor level underneath the formwork for the girders.

Four 3-decker reels similar to the one shown in Fig. 6a held the rolls of prestressing wire. One end of each of these 12 rolls of wire was then passed through a hole in the template shown in Fig. 6b. The holes in the template were arranged in a circle to form the 12 wires in the desired shape. At the center of the circle was a larger hole through which the central helical wire spacer was passed at the same time. The 12 wires and their enclosed spacer were then gripped by the wedge-fitting shown in Fig. 6c which was drawn by a hand winch along the surface of a long narrow table. While the cable was being pulled out in this way, one man standing alongside the template, placed

a single turn of tie wire around the 12 wires at approximately 2 ft. intervals. When drawn out to length, the cable was cut and a steel sheath was then ready for installation in the formwork. This operation is illustrated in Fig. 6d.

Fig. 7 shows the installation of a typical cable in the formwork of which the base and one side is already in place, as are all the precast blocks. Fig. 8 shows a closer view with all the cables in position and the small number of light standard steel stirrups pushed out of the way against each precast block. The next stage was to spread out the stirrups, close the second side of the formwork and place the forms for the recesses in the top flange out of which emerge those cables that do not go right to the end of the girder.

Seven days after pouring of the girder, the first stage of prestressing commenced, and Fig. 9 shows one of the girders with its side forms stripped and one of the first phase cables being tensioned. The hydraulic jack shown is activated through the rubber tubing connected to it, by a hand pump.

Fig. 10 shows a close up of this tensioning operation and the measuring of the elongation of the tensioned wire. Measuring of this elongation is a check on the already registered pressure gage reading on the pump which activates the jack.

Once the correct elongation was obtained, each wire was anchored in the anchor plate by two small conical steel wedges, as seen in Fig. 11 showing the end anchorage plate after the jack has been removed and the wire has been cut. This method, in which the 12 wires are stressed simultaneously by a single jack but are individually anchored, affords greater security against unequal slippage than when the anchorage of a number of wires is dependent on one common wedge. It also affords an opportunity to correct the stress on any wire which has been observed to show greater slippage than its neighbors before the wedges take hold, without having to re-stress the whole cable as is necessary with the use of a single common wedge.

Another problem in stressing a multiple parallel wire cable, particularly a parabolic cable, is that, when jacking is started each wire will show a different strain in adjusting its position in the cable before all the wires start to take uniform stress. This can be serious where the total load in the cable is so unequally distributed in the wires that some are stressed beyond the yield point in the range of high creep loss.

During the work at Manhattanville an interesting experiment was carried out to correct this condition. The large jack, illustrated earlier, was equipped with 12 small jacks around the main cylinder, as shown in Fig. 12, which were connected in series to a separate pump and each engaging a separate wire. The first operation was to actuate the 12 small jacks until the variable strain was taken out of the 12 wires and they were all brought under a uniform stress of around 5,000 p.s.i. It was found that to attain this desirable condition a variation in elongation up to 1 1/2 inch was necessary in the 12 wires. After the individual jacks were locked in position the large jack was brought into play to bring all the wires up to the uniform required stress of 150,000 p.s.i. Unfortunately, this arrangement to insure uniform stress in the wires was only developed at the end of the work, but it represents such an improvement in controlling uniformity of stress that it will be used in all future work of this kind.

After completion of the tensioning of the ten cables in each girder constituting the first phase of prestressing, the slab was poured. A view of the top of the poured slab is seen in Fig. 13 where the as yet untensioned second phase cables are seen projecting from the top flange of the girders.

When the slab was judged to have set and hardened sufficiently, its formwork was removed and its load thus transferred to the girders which now

became ready to receive the second phase of prestressing which was applied as shown in Fig. 14.

After completion of all prestressing the recesses containing the cable anchorages were concreted up and the anchorages at the saw-tooth ends of the girders were concreted over to give a simple rectangular appearance.

A view of the completed girders and slab is seen in Fig. 15.

An interesting discovery made during construction concerns the friction developed in curved cables, many of which had a total curvature of 60°.

Although straight cables were naturally to be tensioned from one end only, it was originally required that all curved cables be tensioned from both ends to minimize stress variations along their length due to friction. Experiments on the site showed, however, that by tensioning even the most curved cable from one end only the desired elongation could be obtained corresponding to the correct reading on the pump gage, after, in the worst case, only a slight relaxing and re-applying of the tension during the stressing operation. This correspondence between the elongation and gage reading was, of course, proof of friction having been overcome along the entire cable length. The explanation of this was undoubtedly that galvanized prestressing wires were being used and that the steel cable sheaths were quite free of rust at the time of their being placed in the forms.

The Manhattanville project afforded an opportunity to demonstrate a new technique of stressing and anchoring multiple parallel wire cables which for the first time assures uniform stress in all wires and positive individual end anchorage. However, had they been obtainable in time for this work we would have preferred to use the high strength steel bars, described by Mr. W. E. Dean, Bridge Engineer of the Florida State Road Department, in his paper on the Tampa Bay Bridge. These bars, soon to be produced in this country, give equal security as to uniform stress and positive anchorage, but with greater economy in cost and simplicity of operation than is possible with any system of parallel wire cables. We are today, in this country, on the threshold of using continuity in design in prestressed structures, which, coupled with more robust and simplified stressing materials and anchorages, will make prestressed concrete competitive with structural steel and reinforced concrete for almost every kind of load-carrying and pressure-containing structure. We have now fortunately passed beyond the development stage in which prestressing was restricted by patents to a few propriatory systems to where we can look to a future in which designers, contractors and material suppliers will be judged solely on the merits of their services and materials in the same competitive spirit as prevails in conventional design.

The Architects for the Manhattanville project were Eggers and Higgins; Weiskopf and Pickworth were the Consulting Engineers, the girders were design by The Preload Company, Inc. and the General Contractor was the George A. Fuller Company.

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